DETERMINATION OF CONTACT PRESSURE AGAINST A LARGE RAFT FOUNDATION

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CHI-YU TENG

B.S., National Central University, 1942
M.S., University of Illinois, 1946

THESIS

SUBMITTED IN PARTIAL FULFILLMENT OF THE REQUIREMENTS
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DETERMINATION OF THE CONTACT PRESSURE AGAINST A LARGE RAFT FOUNDATION

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Introduction

The distribution of the contact pressure against the base of large raft foundations has long been a matter for speculation. Various assumptions have been made in the design of such rafts, of which the most common are that the distribution of the contact pressure is planar, and that the pressure is proportional to the settlement of the structure. Although some experimental evidence has been collected in connection with small loaded areas, no observational data have been available with respect to full-sized raft substructures.

In 1927, a new boiler room was constructed for the Nonoalco Power Plant in Mexico City. The structure is approximately 180 × 68 ft. in plan. The foundation consists of a rectangular gridwork of intersecting steel trusses 14·5 ft. deep resting upon a thin slab of reinforced concrete. The superstructure is of extremely flexible construction. As a consequence, practically all the rigidity of the structure is supplied by the steel truss gridwork.

Accurate differential levels were taken during construction and have been repeated at various intervals up to the present time. These level observations provide the basis for estimating the distribution of contact pressure against the raft. Because of the fortunate coincidence that the superstructure is highly flexible and that the raft consists of structural steel which may be regarded as elastic in its action, it is possible to compute the forces necessary to deform the raft into the positions observed.

The results of the computations are shown in the form of contours of contact pressure. They demonstrate that even after the dead and live loads became constant, the contact pressure continued to change. However, with a fair degree of accuracy, it may be stated that the contact pressure has always been practically uniform beneath most of the base of the structure.

Description of Structure

The site of the present boiler house has been occupied since 1897, when a structure was erected to house six vertical steam engines and six to eight boilers. No data are available concerning the exact location or weight of this equipment, but it has been estimated that the pressure beneath each unit was about 500 pounds per sq. ft. Only a shallow excavation was made and it may be assumed that the subsoil became somewhat consolidated beneath the old equipment.

In 1927, the old boiler house was demolished except for its north wall, and a new one constructed. The north wall was allowed to remain to form a partition between the new boiler house and an existing turbine room along the north side. The excavation for the new raft foundation was made between October 27 and December 24, 1927. Construction of the raft was completed on February 24, 1928, and the full dead load of the completed structure and its equipment was active by October 12, 1928.

Fig. 1 shows a plan of the raft foundation and two sections indicating the layout of the foundation trusses as well as the character of the superstructure. The foundation contains eleven transverse trusses approximately 68 ft. long and five longitudinal trusses approximately 180 ft. long. The general appearance of the structural steel during erection is shown by the photograph, Fig. 2 (Plate II). The detail drawings of all of the trusses were available from which the properties required for computing the load-deflection characteristics could be determined.

The concrete slab beneath the trusses has a thickness of one foot and is reinforced. The slab transfers its load to the trusses through grouted concrete pads located at all of the truss

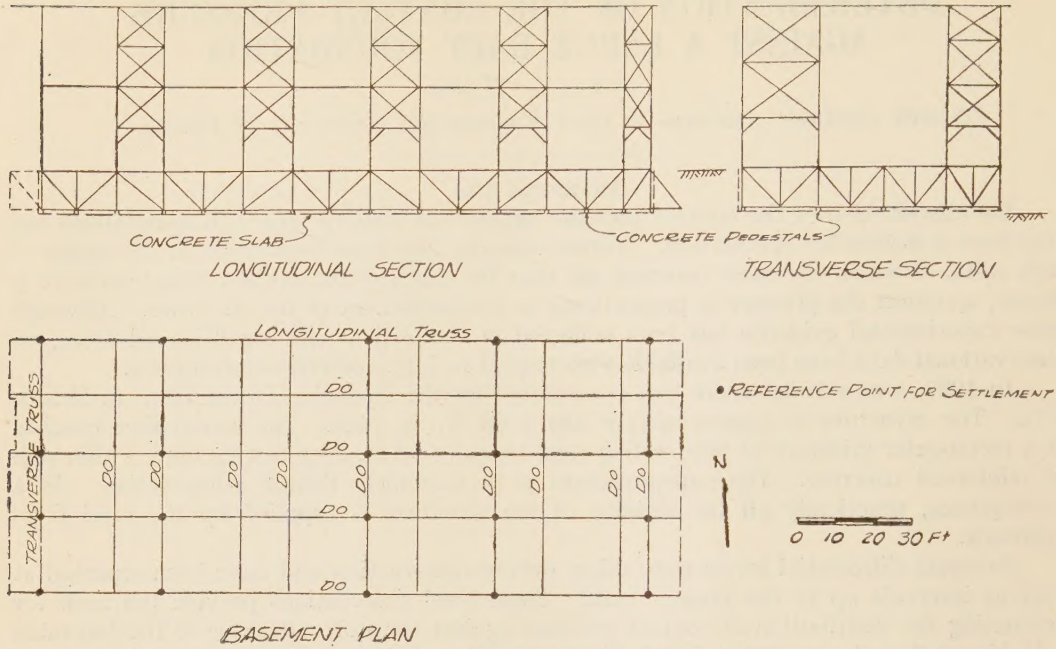


FIG. 1. BOILER HOUSE PLAN AND SECTIONS

intersections and beneath some intermediate panel points, as shown in Fig. 1. The south, east and west sides of the raft also support an 8-inch reinforced concrete retaining wall.

The weights of all parts of the structure and of the equipment are accurately known.

The subsoil of the structure consists of volcanic silty clay to great depth. This material is highly compressible. Its general properties have been described in several papers.* No detailed soil studies were made at the site.

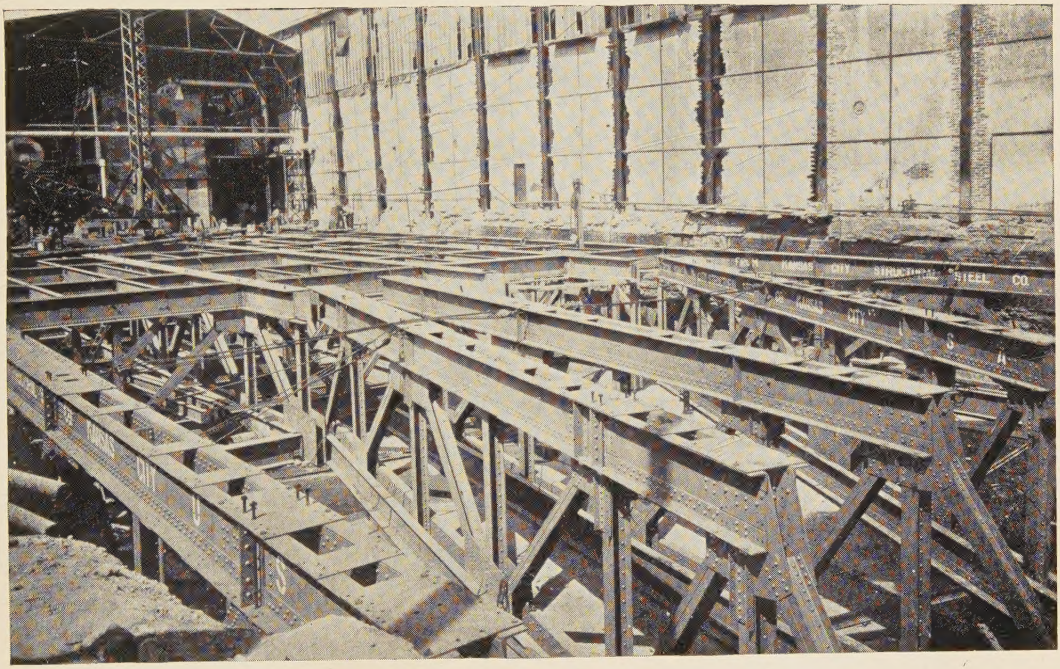
Twenty reference points were established at locations corresponding to the intersection of trusses. These were observed with respect to one set of bench marks from the time of construction until 1938. The bench marks were found to be unstable. As a consequence, the total settlement is unknown. However, the relative settlements among the twenty points are known with an accuracy of approximately 1 mm. In 1938, the original system of reference points was abandoned and a new series established on marks set at a constant elevation. A period of about one month elapsed between the last set of readings on the old reference points and the first set on the new ones, and the two systems were not correlated. Therefore, the settlement record during the last ten years is informative only of the increase in differential settlement since 1938.

METHOD OF ANALYSIS

The method used for determination of the contact pressures included the computation of a consistent system of reactions to the downward loads. These reactions were applied at the concrete pads in such a manner that they would produce the observed deformations of the raft, on the assumptions that the stiffness of the structure was provided entirely by the steel trusses and that the trusses behaved elastically. Subsequently, each reaction was divided by the area enclosed by a set of lines equidistant between the pedestal through which the reaction was transmitted and the adjacent pedestals. To these pressures was added the weight of the concrete slab. The sum was considered the contact pressure.

*A. E. CUMMINGS, "The Foundation Problem in Mexico City," *Proc. Seventh Texas Conf. on Soil Mechanics and Foundation Engineering*, Austin, 1947. L. ZEEVAERT, "The Outline of a Mat Foundation Design on Mexico City Clay," *Proc. Seventh Texas Conf. on Soil Mechanics and Foundation Engineering*, Austin, 1947.

PLATE II



Courtesy of G. D. Camp

FIG. 2. VIEW OF FOUNDATION TRUSSES DURING ERECTION

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The sixteen trusses intersect at fifty-five points. If the relative deflection of each point of intersection were known, the problem would have a unique solution. Unfortunately, only twenty points were observed. Hence, it was necessary to establish some reasonable procedure for estimating the movements of those points on which no observations were made. This was done by constructing curves of equal differential settlement for the observed points and noting their general pattern. Deflection curves were then drawn for the trusses in such a way that the unobserved points had reasonable positions and did not create unusual changes in the pattern of the settlement contours. By this means, settlements were assigned to all the unobserved points of intersection. The computation of reactions could then be carried out by methods of ordinary structural analysis which, although tedious, presented no unusual difficulties. The computed reactions satisfy the requirements that they equal the total vertical downward load on the soil and that the centre of gravity of the applied loads coincides with that of the contact pressure.

After the computations were made on this basis, the influence of the several sources of error was investigated. Among these were the effect of the 8 in. reinforced concrete walls on three sides of the structure, the stiffness contributed by the wind bracing in the superstructure, the effect of shearing forces in the pads between the base slab and the trusses, and the influence of possible shearing stresses in the subsoil beneath the mat. It was found that none of these was likely to produce an error of more than a few per cent and that the combined error was unlikely to exceed about 10 per cent of the value of the maximum contact pressure.

Additional studies were carried out to determine the error in contact pressure inherent in the interpolation procedure. For this purpose, several sets of contact pressures were computed using interpolated deflections believed to be the most extreme values at all reasonable. It was found that the error in contact pressure due to possible differences between the actual and interpolated values is probably less than 10 per cent, except along the north edge of the raft where the error may be considerably greater.

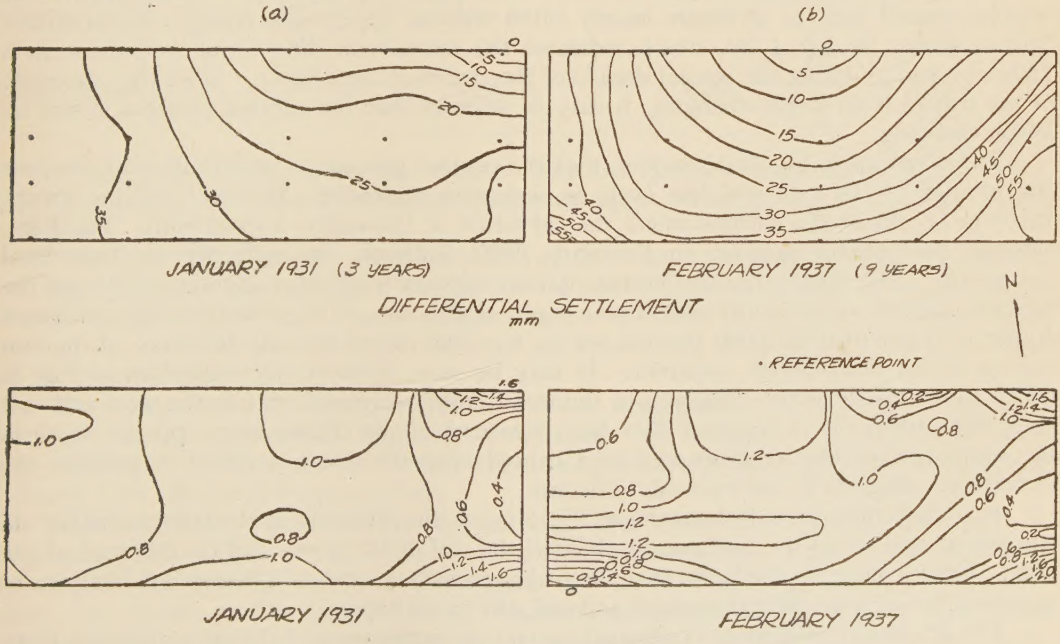

FIG. 3. DIFFERENTIAL SETTLEMENT AND CORRESPONDING CONTACT PRESSURE ON THE BASE OF THE RAFT AT TWO DIFFERENT TIMES

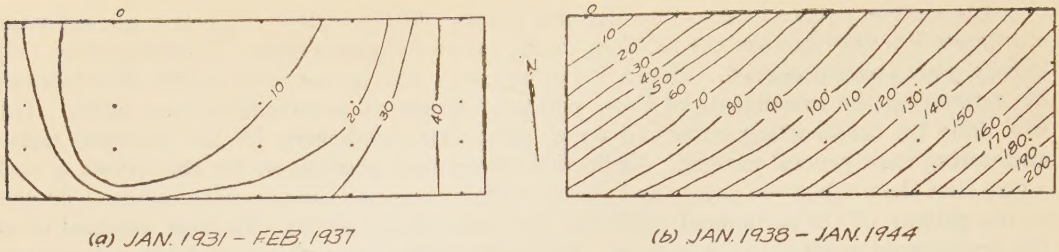

FIG. 4. INCREASE IN DIFFERENTIAL SETTLEMENTS IN TWO 6-YEAR PERIODS

Hence, it may be concluded that the contact pressures computed by the procedures described are a sufficiently accurate representation of the real pressures.

RESULTS

Fig. 3 (a) shows contours of differential settlement three years after completion of the structure together with the corresponding computed contact pressure. Similar information is shown in Fig. 3 (b) for a time nine years after completion. It may be observed that the stiff raft was successful in reducing differential settlements to an inconsequential amount, considering the fact that unstiffened buildings of this size in Mexico City commonly experience a differential settlement of the order of several feet. Therefore, the raft may be considered a very satisfactory foundation.

In the first ten years, the structure gradually warped into a half-dome shape with the least settlement near the centre of the north wall and the most at the south corners. This is shown more clearly in Fig. 4 (a) which indicates the increase in differential settlement in a 73-month period during the first decade of the life of the structure. Subsequently the warping ceased and the structure merely tilted without appreciable changes in curvature. This is shown by Fig 4 (b), which indicates the increase in differential settlement in a 72-month period during the second decade of the structure's existence. Since the curvature of the trusses is no longer changing, it may be inferred that the contact pressure is also no longer changing.

The diagrams in Fig. 3 (b) suggest that the contact pressure is essentially uniform over the interior of the raft and has been so since the beginning. However, rather abrupt variations occur at the corners where the curvature of the raft is a maximum. Fig 3 (b), showing the contact pressure in February, 1937, indicates the influence of these local curvatures. The differential settlement pattern appears to be fairly symmetrical, but the contact pressure at the south-west corner is practically zero, whereas that at the south-east corner is approximately 2,000 pounds per sq. ft. The reason for this difference at the two corners is not immediately apparent. It may be seen, however, by inspection of Fig. 5, which shows the deflection diagrams of the two transverse trusses next to the west and east ends respectively. It is apparent that the curvatures of the trusses have opposite sense in the south half of the structure and as a consequence the forces required to produce the curvatures (diagram b) are radically different.

It is likely that the half-dome shape, Fig. 3 (a), is due primarily to the influence of the old equipment and structure that preconsolidated the soil in the north and central part of the area. Thus it may be concluded that the real variations in contact pressure are likely to be governed by such accidental features as local stiff or soft spots.

The practical implications of the real contact pressures are perhaps most apparent if the maximum stresses in the various members of the raft are compared with those computed on the basis of various assumptions. Such a comparison is made in Tables I and II, which refer to the longitudinal and transverse trusses respectively. Each table indicates the

TABLE I
MAXIMUM UNIT STRESSES IN LONGITUDINAL TRUSSES
KIPS PER SQUARE INCH

Member	Assumption Used by Designers	Assumed Uniform Soil Pressure	Computed from Deflections
Verticals	7·8C	12·2C	4·0C 6·1T
Diagonals	10·8C 11.9T	7·7C 1·3T	11·1C 10·1T
Upper Chords	9·5C	2·2C	0·9C 5·4T
Lower Chords	15·2T	3·4T	8·3C 1·9T

TABLE II
MAXIMUM UNIT STRESSES IN TRANSVERSE TRUSSES
KIPS PER SQUARE INCH

Member	Assumption Used by Designers	Assumed Uniform Soil Pressure	Computed from Deflections
Verticals	10.7C	22.6C	26·7C 17.3T
Diagonals	8.5C 10·5T	0.2C 5·3T	20.3C 20·3T
Upper Chords	5·2C	1·6C	6·7C 2·9T
Lower Chords	4·5T	1·3T	6·3C 7·9T

stress, in kips per square inch, for whatever member is most highly stressed. The comparison is made separately for verticals, diagonals, upper chord members, and lower chord members.

The first values shown are those computed on the basis of the assumptions actually made by the designers. The assumed distribution of contact pressure, Fig. 6, was probably chosen to approximate that beneath a rigid plate on an elastic foundation. The stresses are very moderate, with a maximum of 15·2 kips per square inch in some lower chord members of the longitudinal trusses.

The values shown in the next column are those computed on the common assumption that the contact pressure is uniformly distributed. As would be expected, the chord stresses are very small, whereas the stresses in some of the web members are greater than those computed on the previous assumption.

The last column indicates the stresses computed from the observed curvatures in February, 1937. They are probably a reasonable representation of the actual maximum stresses. A few generalizations seem warranted on the basis of these values. In the first place, the stresses in the long, relatively flexible longitudinal trusses are less than those computed in accordance with either set of assumptions, whereas those in the short, relatively stiff transverse trusses are generally greater. This would be expected on the basis of the comparative rigidities of the members of the two systems.

Chord stresses (i.e. bending moments) estimated on the assumption of a uniform soil

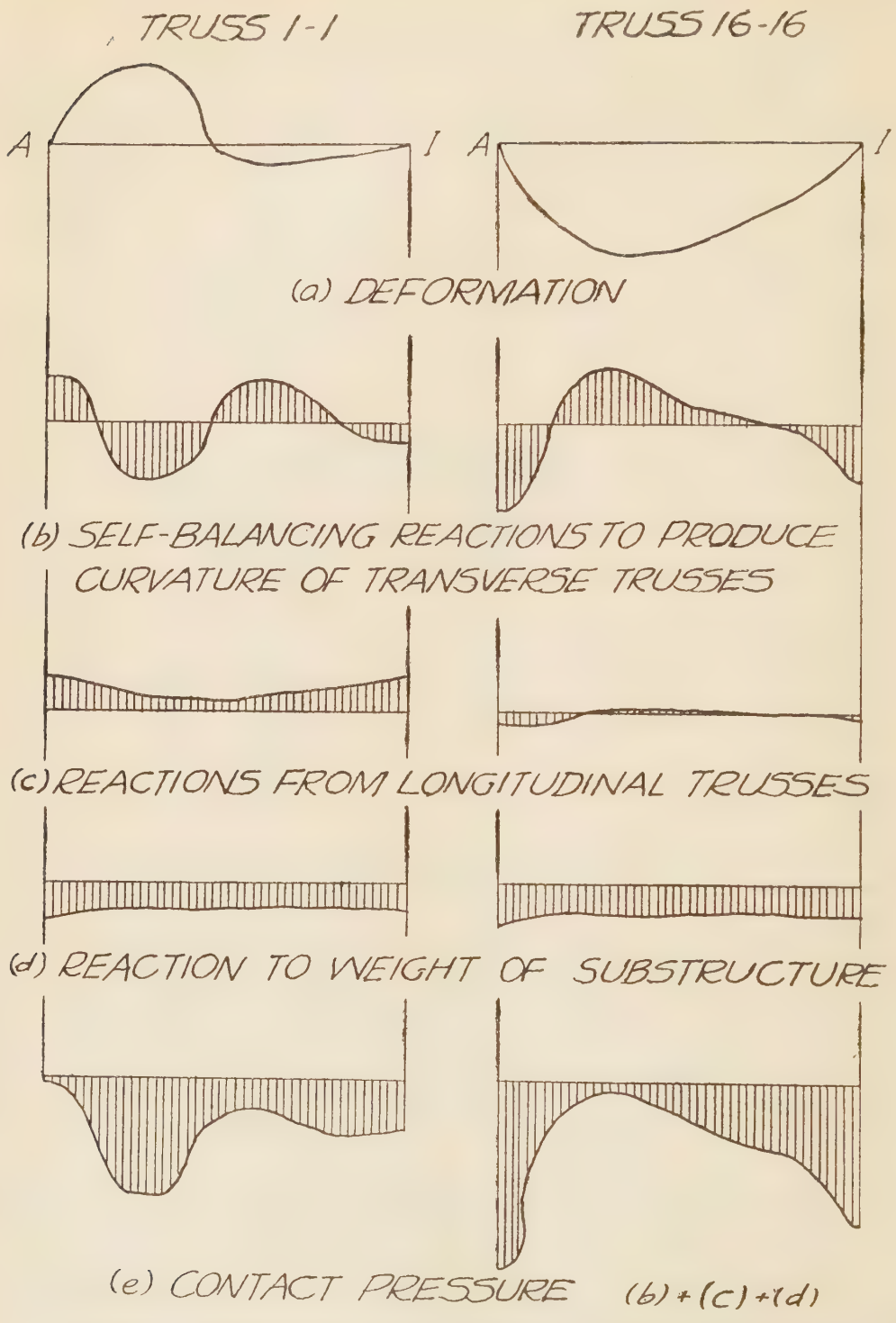

FIG. 5. DIAGRAMS SHOWING VARIATION IN CONTACT PRESSURE DUE TO DIFFERENCES IN CURVATURE OF TRANSVERSE TRUSSES

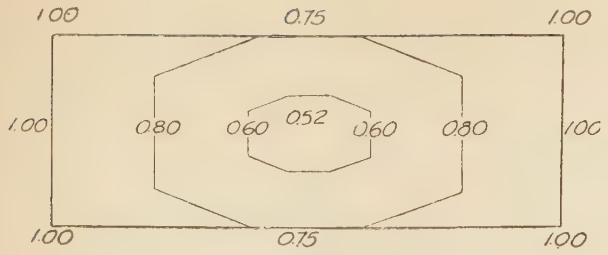

Fig. 6. Contact Pressure Assumed by the Designers Kips per Sq. Ft.

pressure are much too low, because the curvatures due to consolidation of the clay and to the influence of local soft and stiff spots are neglected. The pressure distribution assumed by the designers afforded a reasonable means of taking these curvatures into account and led to a satisfactory structure, in spite of the fact that the signs of the chord stresses were generally opposite to those computed because the settlement of the structure had the shape of a dome rather than a bowl. Web stresses (i.e. shears), on the other hand, would have been estimated more conservatively on the assumption of a uniform contact pressure.

These remarks suggest that the contact pressure may be largely a function of unpredictable variations in the compressibility of the subsoil and that it cannot be determined with reasonable accuracy on theoretical grounds. It would seem more appropriate to assume several limiting conditions representing reasonable variations from a uniform distribution of pressure, and to design each member of a raft for the most severe stresses indicated by any of the assumed conditions.

On the basis of the studies of the Nonoalco Plant, which probably represent the only observational data at our disposal concerning the actual contact pressure beneath a large raft, it seems apparent that judgment is destined to remain the prime requisite for the design of large rafts. Proper evaluation of the influence of variations in the stiffness of the soil and the structure appears to be more important than reliance on mathematical procedures, such as the theory of subgrade reaction.

Acknowledgment

This paper is abstracted from a thesis constituting part of the requirements for the degree of Ph.D. in Engineering at the University of Illinois. The topic was suggested by Dr. Karl Terzaghi who had previously obtained most of the factual data concerning the structure through the courtesy of the Mexican Light and Power Company. The structure was designed by Sargent and Lundy, of Chicago.

The thesis was under the direction of Professor Ralph B. Peck. Assistance in developing the method of computation was given by Professor N. M. Newmark.

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